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MMM Group Ltd.
Rivergate Outfall

Geotechnical Investigation Report
August 2012

Our File No. 0080 001 00

Prepared for:

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Winnipeg, Manitoba
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APEGM
Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: Aug 27, 2012

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1.0 Introduction

TREK Geotechnical Inc. (TREK) was retained by MMM Group Ltd. (MMM) to provide geotechnical design recommendations for proposed upgrades to the Rivergate Outfall that is located on the east bank of the Red River on South St. Mary's Road. The scope of work undertaken for the project was approved by MMM on June 28, 2012. This report summarizes the results of the work completed and provides geotechnical recommendations for design and construction of the outfall upgrades.

TREK's scope included the following specific tasks:

- Review existing information about the site,
- Complete a sub-surface investigation consisting of one borehole at the top of bank near the existing outfall,
- Complete a cross-sectional survey of the riverbank along with other relevant features.
- Prepare a final report incorporating recommendations on slope stability, foundation requirements, shoring for temporary excavations and basal heave during construction of the new gate chamber.

This report represents the completion of the scope of work agreed to in our Engineering Services contract dated and signed by MMM on June 28, 2012.

2.0 Project Understanding

The existing gate chamber for the outfall will be replaced with a new chamber near the top of bank area. It is understood that the new gate chamber will have dimensions of 10 m x 6 m and the base of the footing will extend down to El. 220.4 m.

3.0 Exploration Program

One test hole was drilled near the existing gate chamber to determine the soil stratigraphy and groundwater conditions at the site. The drilling was completed on July 12, 2012 by Paddock Drilling under the supervision of TREK personnel. The test hole was drilled using a 125 mm solid stem auger mounted on a Mobile RM-30 track-mounted drill rig. Subsurface conditions observed during drilling were visually classified using the Unified Soil Classification System (USCS). Relatively undisturbed (Shelby tube) and disturbed (auger cutting) samples were recovered in cohesive soils. Standard penetration testing (SPT) was also performed within the glacial till layer. Other pertinent information such as drilling, groundwater and backfill conditions was also recorded. On completion of drilling a standpipe piezometer was installed near the bottom of the hole to evaluate groundwater pressures in the glacial till. Samples retrieved during drilling were transported to TREK's testing laboratory in Winnipeg, Manitoba. The laboratory testing consisted of moisture content determination, undrained shear strength testing (pocket penetrometer, Torvane and unconfined compression) and bulk unit weight determination. A copy of the laboratory test results is included in Appendix A.

The test hole was logged continuously to determine soil stratigraphy, obtain soil strength parameters and establish auger refusal depth. A test hole log has been prepared and is included in Appendix A and includes a description of location and the depth of soil units encountered and other pertinent information such as sloughing and groundwater seepage. The location of the test hole is shown in Figure 01.

A survey was also performed on the site. The primary purpose of the survey was to determine the topographical cross-section along the outfall location including depth soundings in the river. The surveyed cross-section is shown in plan and cross-section in Figure 01.

3.1 Sub-surface Conditions

3.2.1 Soil Stratigraphy

The soil stratigraphy observed in the one borehole drilled at the site is summarized below in descending order from ground surface:

- Organic silt (topsoil)
- Silt
- Lacustrine silty clay
- Glacial silt till

A brief description of the soil units encountered during drilling is provided below.

Organic Silt (topsoil)

The organic silt (topsoil) is 0.1 m thick, black and dry. The silt contains some clay, some rootlets, and trace sand.

Silt

Silt was observed from 0.1 to 1.4 m. The silt is dry, dark brown, and firm. The silt contains some clay, trace sand, trace gravel, and trace organics. Below about 0.8 m depth the silt becomes light brown and contains no organics.

Lacustrine Silty Clay

A highly plastic, lacustrine, silty clay layer was encountered beneath the silt and extended to a depth of about 13.7 m (El. 218.4 m). The clay is mottled grey and brown immediately below the silt transitioning to grey below 6.7 m depth. Moisture contents in the clay generally increase with depth ranging from 38% to about 70%. Measured bulk unit weights range from 16.9 to 17.9 kN/m^3. Atterberg limits were performed on two samples of the clay and resulted in an average liquid limit of approximately 85% and an average plastic limit of approximately 23%. The profile of undrained shear strength shown on the test hole log indicates a thin upper crust of stiff clay to a depth of about

2 m below which the clay is firm to soft with increasing depth. Average undrained shear strengths range from about 100 kPa at a depth of 2 m decreasing to about 40 kPa below a depth of 2.5m. The undrained shear strength of the clay continues to decrease gradually with depth to about 25 kPa just above the till contact.

Glacial Silt Till

Silt till was encountered beneath the clay layer at a depth of 13.7 m. The till is sandy and contains trace to some rounded to subrounded gravel, trace to some clay and is light grey, moist and loose. At 16.2 m depth the till becomes light brown, dry to moist, and very dense.

3.2.2 Groundwater

Seepage and sloughing was encountered within the silt till unit below 15.1 m. A standpipe piezometer was installed within the till and groundwater elevations of 223.9 m and 223.7 m were observed in the standpipe one week and three weeks after installation respectively. It is likely that groundwater levels within the till at the test hole location are influenced by water levels in the Red River (surveyed water elevation of 223.8 m on the day of drilling), however this would need to be confirmed through seasonal monitoring. It is also important to note that groundwater conditions may change seasonally, annually, or as a result of construction activities.

3.2.3 Auger Refusal

Auger refusal within the glacial silt till was reached at a depth of 16.4 m (El. 215.8 m). Although samples could not be recovered from the test hole terminus, it is suspected that auger refusal was on dense glacial till.

4.0 Geotechnical Recommendations

4.1 Riverbank Stability

The proposed upgrading works are not expected to negatively impact the existing riverbank stability as there will be no increase in loading at the top of the bank in the vicinity of the gate chamber. Stability analysis was carried out however, to confirm that there is an adequate level of bank stability in the immediate vicinity of the proposed works both during construction (short term) and in the longer term. In this regard, we would consider a minimum factor of safety (FS) of 1.3 is appropriate for both short and long term conditions assuming worst case (extreme) groundwater and river levels. Under normal (typical) conditions, a FS of 1.5 would be considered appropriate at the river side of the gate chamber.

The stability analysis was conducted using a limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2007 software package (Geo-Slope International Inc.). The cross section geometry used in the model is based on survey information performed in July of 2012 and the stratigraphic profile is based on the results of the borehole drilled in the top of bank area. Both circular and noncircular (composite) potential slip surfaces were analyzed under what we consider to be worst case groundwater/river levels under summer and winter conditions:

Worst Case Winter: Groundwater level in the clay 2.0 m below surface and groundwater level within the glacial till coincident with the river level at El. 222.0 m.

Worst Case Summer: Saturated bank conditions in the clay and a groundwater level within the glacial till coincident with the river level at El. 223.8 m.

The soil properties used in the slope stability model are presented in Table 1. Unit weights are based on laboratory testing results. The lacustrine clay shear strengths are based on local experience and reflect large strain values since there is no visual evidence of slope failures that would necessitate the use of residual strengths in the analysis. The glacial till strengths are based on local experience and the limestone bedrock is considered impenetrable.

Table 1 - Soil Properties Used in Slope Stability Analysis

Soil Description	**Unit Weight (kN/m^3)**	**Cohesion (kPa)**	**Friction Angle (deg)**
Lacustrine Clay (Large strain strength envelope)	17	5	14
Glacial Silt Till	20	10 (circular slips)	30 (circular slips)
		Impenetrable (composite slips)	
Limestone Bedrock	Impenetrable		

The calculated factors of safety for a potential slip surface at the west (river) side of the proposed shoring for the assumed winter condition was 1.78, for the summer condition the safety factor was 1.59 as shown on Figure 02, in both cases the composite slip surface (glacial silt till impenetrable) provided lower factors of safety. This factor of safety exceeds 1.5 under worst case conditions and additional analysis to evaluate typical conditions is not considered necessary. The minimum calculated FS is 1.31 associated with a potential (theoretical) slip surface entering the slope approximately 25 m west (downslope) of the proposed shoring location.

4.2 Shoreline Erosion

As shown in the photos in Appendix B the shoreline near the outfall is experiencing moderate erosion at this time, and left unattended, may result in a reduction in riverbank stability. To guard against continued shoreline erosion, the placement of a riprap blanket should be considered at the time of the gate chamber work. The riprap blanket would be 0.75 m thick and extend from about El. 225 m to below the winter river level (El. 222 m). Preliminary stability analysis has confirmed that the riprap will not adversely impact on slope stability. Trek can provide detailed recommendations for design of the riprap blanket if desired however; this was not within the scope of the present study.

5.0 Excavations and Shoring

It is understood that an excavation depth of about 11.5 m is required to construct the new gate chamber and that shoring for the excavation will be required.

5.1 Temporary Excavations

Based on the above excavation depth, conventional shoring will need to be braced or tied back. The earth pressure distributions provided in Figure 03 can be used for shoring design using a bulk unit weight of 17.0 kN/m^3 and an active earth pressure coefficient of 0.6. An undrained shear strength of 30 kPa for the clay can be used for the design of shoring and the determination of an adequate factor of safety against toe instabilities. The undrained shear strengths were selected based on the measured undrained shear strength profile from all test types. The effect of any surcharge loads must be added to the force on the wall in addition to the calculated earth pressures. The appropriate earth pressure condition should be used to calculate the lateral earth pressure due to surcharge loads.

Ground movements behind the shoring and associated settlement are largely unavoidable. The amount of movement cannot be predicted with a high degree of accuracy as it is as much a function of the excavation procedures and workmanship as it is of theoretical considerations. In this regard, good contact between the timber lagging and retained soil should be maintained throughout the construction process. Free draining sand fill should be used to fill in any voids behind the lagging. Additional recommendations can be provided should infrastructure sensitive to settlement exist in close proximity to the excavation.

It is anticipated that the design of excavation slopes and temporary shoring will be the responsibility of the Contractor. Shoring designs or excavations greater than 3 m in height will need to be designed and sealed by a professional engineer and reviewed by TREK Geotechnical prior to construction to confirm the parameters and soil conditions used in design are consistent with the recommendations provided herein.

5.2 Groundwater Considerations

The lacustrine clay is underlain by a layer of glacial till under confined groundwater pressures. As a result, the potential for base heave and/or groundwater seepage into excavations must be considered. If base heave occurs causing hydraulic fracturing of the clay, there exists a potential for groundwater seepage into the excavation. This event could be sudden and catastrophic in nature. In this regard, sufficient resisting forces are required to counteract groundwater pressures. The resisting forces are a function of the thickness and unit weight of the clay above the till and, to a lesser degree, shoring dimensions.

An adequate factor of safety against base heave is achieved when the groundwater level in the till is at or below El. 220.8 m for the proposed excavation depth (El. 220.4 m) and excavation dimensions. In comparison, a groundwater elevation of 223.9 m was measured in the glacial till in July 2012. As described in Section 3.2.2 it is anticipated that water levels in the till fluctuate with river levels however this cannot be confirmed at this time. If this is the case, the groundwater levels in the till may be closer to El. 222.0 m during the winter when construction is anticipated, however this elevation is still higher than the groundwater level necessary to achieve an adequate factor of safety. It must also be recognized that groundwater levels are likely to increase during spring freshet before returning to normal summer levels.

The current and future groundwater levels in the till are higher than the level necessary to maintain an adequate level of stability against base heave. In this regard, a relief well (or multiple relief wells) are recommended to depressurize the till layer and achieve an adequate factor of safety against basal heave. The well(s) should be able to maintain the recommended safe groundwater elevation for the length of time the excavation is open and be able to accommodate fluctuations in groundwater elevations. It is important to note that even with an adequate factor of safety against base heave, the potential for seepage into the excavation exists when the groundwater elevation is above the base of the excavation. Should this occur, it will be necessary to dewater the excavation and/or lower the groundwater to an elevation lower than the base. The number and size of well necessary to lower groundwater levels will depend on the permeability and thickness of the till which can be highly variable. It is recommended that a pump test be performed by a qualified hydrogeologist to determine the necessary pumping requirements. As a part of this test, groundwater quality should be evaluated to determine if it is acceptable to discharge directly into the river.

The recommended safe groundwater elevation is a function of excavation depth and shoring dimensions and should be re-calculated for any base elevation other than El. 220.4 m or if there are changes in shoring dimensions. As it appears likely that the glacial till daylights into the river channel at this location, depressurization of the glacial till may require significant effort. The effort required will primarily be a function of the till permeability which can be highly variable.

5.3 Earth Pressures Against New Gate Chamber

TREK understands that the gate chamber excavation outside of the structure is to be backfilled for its full depth (D) using Type 2 fill (as per City of Winnipeg Specification CW2030) compacted to at least 95% Standard Proctor Maximum Dry Density (SPMDD). It is also our understanding that the distance between the shoring and the gate chamber walls will be approximately one metre. It should be recognized that lateral earth pressures induced by compaction against rigid structural walls may be greater than the at-rest pressure and earth pressure coefficients of 1.0 or higher are possible. The earth pressure coefficient is difficult to predict as it depends on several factors including the type, geometry and moisture content of the backfill material and the compactive effort applied. In this regard, it is generally recommended to lightly compact (in the order of 92% of SPMDD) the backfill in close proximity to buried walls unless a higher degree of compaction is necessary *e.g.* for pipe bedding or where minimizing surface settlement is required. Compensation for any settlement can be made in the final grading to provide positive drainage away from the structure. We estimate that backfill compacted in this manner (lightly) will ultimately settle by a maximum of about 2% of the fill depth. For the upper 0.6 m, clay backfill soils should be used to create a low-permeability cap.

Based on the limited space available between the shoring and structure, lateral pressures against the gate chamber are expected to be governed by the properties of the surrounding clay soil. Based on this configuration, we also anticipate that a small vibratory plate compactor will be used for compaction. If this is the case, then earth pressures against the wall can be calculated using a triangular pressure distribution according to the following equation:

$$\mathbf{P = k\gamma D}$$

Where P = lateral earth pressure at depth D (kPa)
k = earth pressure coefficient (0.7)
γ = unit weight (17 kN/m^3)
D = depth from surface to the point of the pressure calculation (m)

Lateral earth pressures from surcharge loads (if applicable), or for heavy compaction equipment (if used) should be accounted for in design, TREK can provide recommendations for loading should they be needed. If drainage is not provided at the base of the gate chamber, the buoyant soil unit weight should be used and the water (hydrostatic) pressure added assuming a water level coincident with the ground surface. The shoring geometry, backfill types and compaction methods should be reviewed during final design.

6.0 Foundations

Structures of this nature are often supported by a mat foundation buried deep into the soil where part (or all) of the loads may be compensated by the weight of removed soil. Based on the design elevation provided by MMM (El. 220.4 m), foundation soils are expected to consist of soft to firm lacustrine clays. A maximum allowable bearing capacity of 250 kPa is recommended for the design of foundations on the clay at the proposed elevation of 220.4 m. It should be noted that this bearing capacity is based on a maximum estimated settlement of 25mm at the maximum allowable bearing pressure. Should such settlement not be acceptable, a deep foundation system consisting of either cast-in-place or driven piles could be considered. Aside from consolidation settlement, vertical displacements of the structure can ensue if changes in the moisture content of the clay occur during construction, in this case drying. Measures to minimize the drying potential, for example a mud slab, may be considered. Uplift (buoyant) forces acting against the access chamber should also be considered in design and a groundwater level at existing ground surface should be used.

7.0 Waterways Permit

A Waterways Permit from the City of Winnipeg is required to carry out the work. It is expected that conditions of the permit are likely to include the stockpiling of materials well away from the top of bank and written right-of-access from any adjacent property owners where access may be required. It will also be necessary to restore any access or egress routes in the same or better condition than before construction. The Waterway Permit application should therefore include any proposed access and egress routes and stockpile locations.

Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings and recommendations of this report were based on information provided (field investigations, laboratory testing, geometries, equipment specifications) and interpolation of soil and groundwater information between test holes. Soil conditions are natural deposits that can be highly variable across a site. If sub-surface conditions are different than the conditions previously encountered on-site or those presented here, or if the assumptions presented in this report are not keeping with the overall design or construction procedures, we should be notified to review our recommendations and adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

Figures

TREK GEOTECHNICAL

Rivergate Outfall Geotechnical Investigation
-
Winnipeg, Manitoba

ANSI full bleed B (11.00 x 17.00 Inches)

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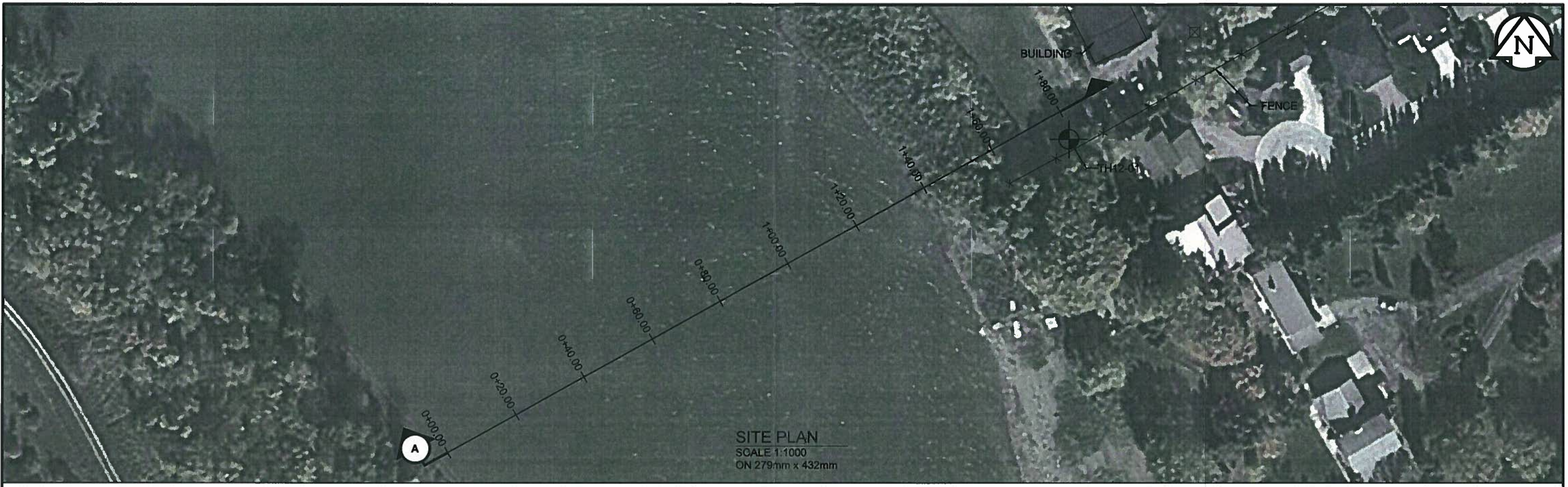

SITE PLAN
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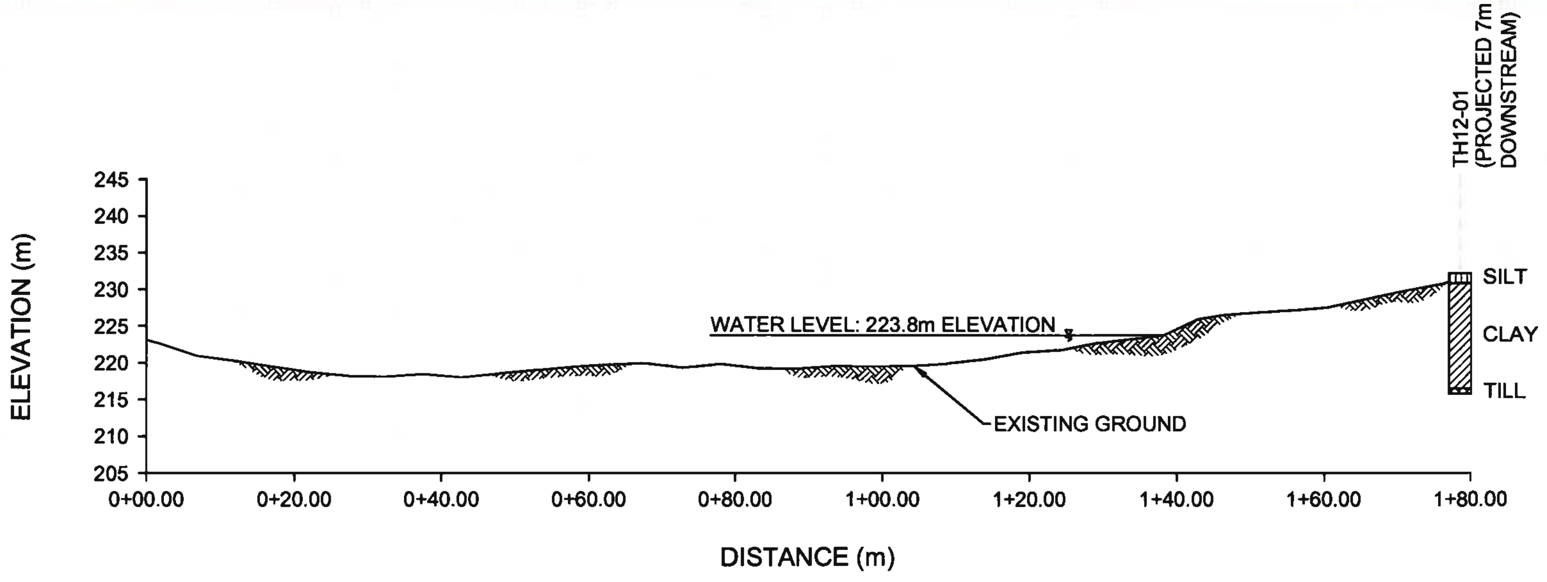
ON 279mm x 432mm



CROSS-SECTION A
SCALE 1:1000
ON 279mm x 432mm

0 10 20 30 40m
SCALE : 1:1000 (279mm x 432mm)

LEGEND :

- ⌖ TEST HOLE
- ☒ MAN HOLE

Figure 01
Site Plan and Cross-Section A

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Rivergate Outfall Geotechnical Investigation

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Winnipeg, Manitoba

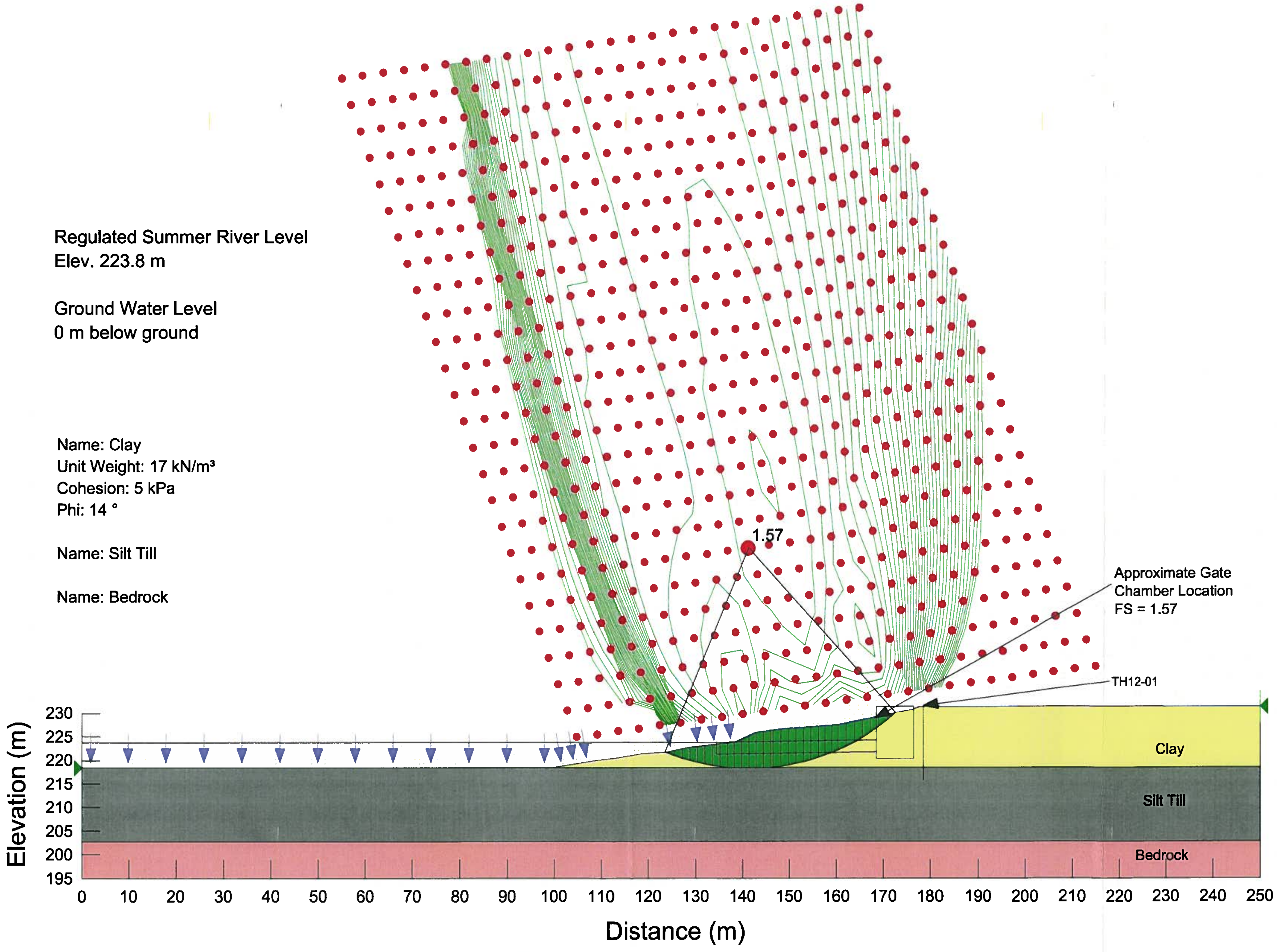

Figure 02
Slope/W Model

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MMM Group
Rivergate Outfall
Winnipeg, Manitoba

8 1/2" x 11"

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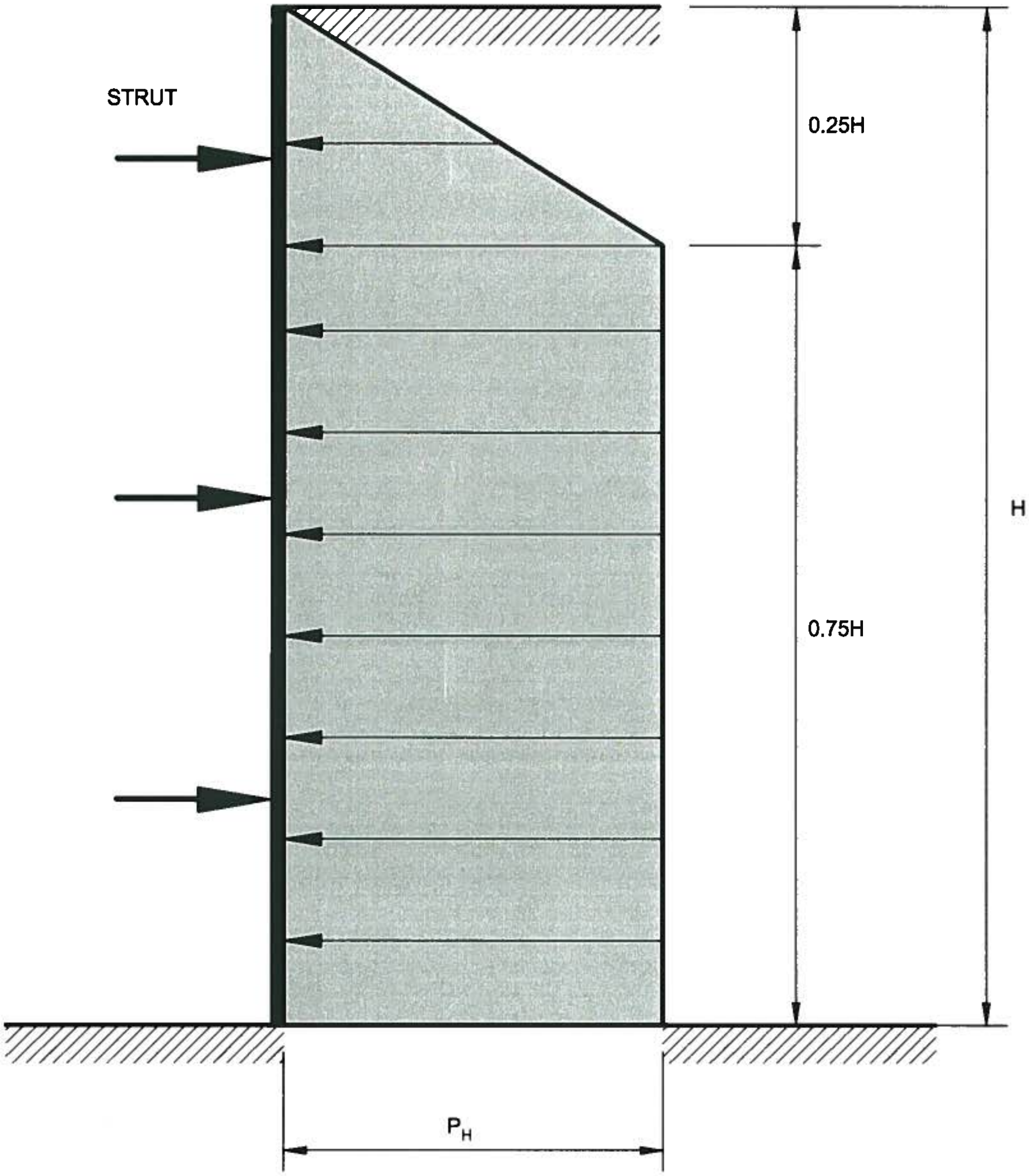


$P_H = 0.6\,\gamma\,H$

WHERE :

H = DEPTH OF EXCAVATION (m)

P_H = LATERAL EARTH PRESSURE (kP_a)

γ = BULK SOIL UNIT WEIGHT $(17.0\ kN/m^3)$

NOTE:

- ADD SURFACE LOAD SURCHARGE IF APPLICABLE

Lateral Earth Pressure Distributions
Braced Excavations in Clay
Figure 03

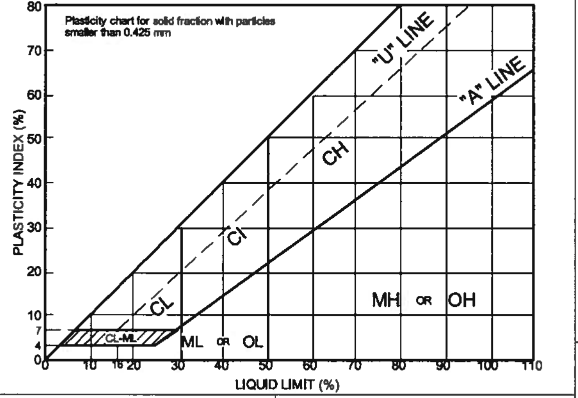
Appendix A

Test Hole Log

EXPLANATION OF FIELD AND LABORATORY TESTING

GENERAL NOTES

1. Classifications are based on the United Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
2. Descriptions on these test hole logs apply only at the specific test hole locations and at the time the test holes were drilled. Variability of soil and groundwater conditions may exist between test hole locations.
3. When the following classification terms are used in this report or test hole logs, the primary and secondary soil fractions may be visually estimated.

Major Divisions			USCS Classi-fication	Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than 4.75 mm)	Clean gravel (Little or no fines)	GW	◙	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent....... GW, GP, SW, SP More than 12 percent...... GM, GC, SM, SC 6 to 12 percent..... Borderline case4s requiring dual symbols*	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	○	Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravel with fines (Appreciable amount of fines)	GM	▥	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			GC	▨	Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than 4.75 mm)	Clean sands (Little or no fines)	SW	⁘	Well-graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	░	Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM	▥	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			SC	▨	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-Grained soils (More than half the material is smaller than No. 200 sieve size)	Silts and Clays (Liquid limit less than 50)		ML	▥	Inorganic silts and very fine sands, rock floor, silty or clayey fine sands or clayey silts with slight plasticity	Plasticity Chart 		
			CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	☰	Organic silts and organic silty clays of low plasticity			
	Silts and Clays (Liquid limit greater than 50)		MH	▮	Inorganic silts, micaceous or distomaceous fine sandy or silty soils, organic silts			
			CH	▨	Inorganic clays of high plasticity, fat clays			
			OH	≋	Organic clays of medium to high plasticity, organic silts			
	Highly Organic Soils		Pt	⁂	Peat and other highly organic soils	Von Post Classification Limit	Strong colour or odour, and often fibrous texture	

Material		Particle Size	
		mm	ASTM Sieve sizes
Sand	Coarse	2.00 to 4.75	#10 to #4
	Medium	0.425 to 2.00	#40 to #10
	Fine	0.075 to 0.425	#200 to #40
Silt or Clay		< 0.075	< #200

Material		Particle Size	
		mm	ASTM Sieve Sizes
Boulders		> 300	> 12 in.
Cobbles		75 to 300	3 in. to 12 in.
Gravel	Coarse	19 to 75	3/4 in. to 3 in.
	Fine	4.75 to 19	#4 to 3/4 in.

* Borderline classifications used for soils possessing characteristics of two groups are designated by combinations of groups symbols. For example; GW-GC, well-graded gravel-sand mixture with clay binder.

Other Symbol Types

■	Asphalt	▧	Bedrock (undifferentiated)	○	Cobbles
⁘	Concrete	⊞	Limestone Bedrock	◙	Boulders and Cobbles
▩	Fill	☰	Cemented Shale	▥	Silt Till
		▤	Non-Cemented Shale	▨	Clay Till

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EXPLANATION OF FIELD AND LABORATORY TESTING

LEGEND OF ABBREVIATIONS AND SYMBOLS

LL - Liquid Limit (%)
PL - Plastic Limit (%)
PI - Plasticity Index (%)
MC - Moisture Content (%)
SPT - Standard Penetration Test
RQD - Rock Quality Designation
Qu - Unconfined Compression
Su - Undrained Shear Strength
VW - Vibrating Wire Piezometer
SI - Slope Inclinometer

▽ Water Level at Time of Drilling

▼ Water Level at End of Drilling

▼ Water Level After Drilling as Indicated on Test Hole Logs

FRACTION OF SECONDARY SOIL CONSTITUENTS ARE BASED ON THE FOLLOWING TERMINOLOGY

TERM	EXAMPLES	PERCENTAGE
and	and CLAY	35 to 50 percent
"y" or "ey"	clayey, silty	20 to 35 percent
some	some silt	10 to 20 percent
trace	trace gravel	1 to 10 percent

TERMS DESCRIBING CONSISTENCY OR COMPACTION CONDITION

The Standard Penetration Test blow count (N) of a non-cohesive soil can be related to compactness condition as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very loose	< 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	> 50

The Standard Penetration Test blow count (N) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	SPT (N) (Blows/300 mm)
Very soft	< 2
Soft	2 to 4
Firm	4 to 8
Stiff	8 to 15
Very stiff	15 to 30
Hard	> 30

The undrained shear strength (Su) of a cohesive soil can be related to its consistency as follows:

Descriptive Terms	Undrained Shear Strength (kPa)
Very soft	< 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	> 200

Sub-Surface Log

Test Hole TH12-01

Client: MMM Group Limited

Project Number: 0080 001

Project Name: Rivergate Outfall Geotechnical Investigation

Location: UTM 14 N-5519799.701, E-635635.622

Contractor: Paddock Drilling Ltd.

Ground Elevation: 232.15 m Existing Ground

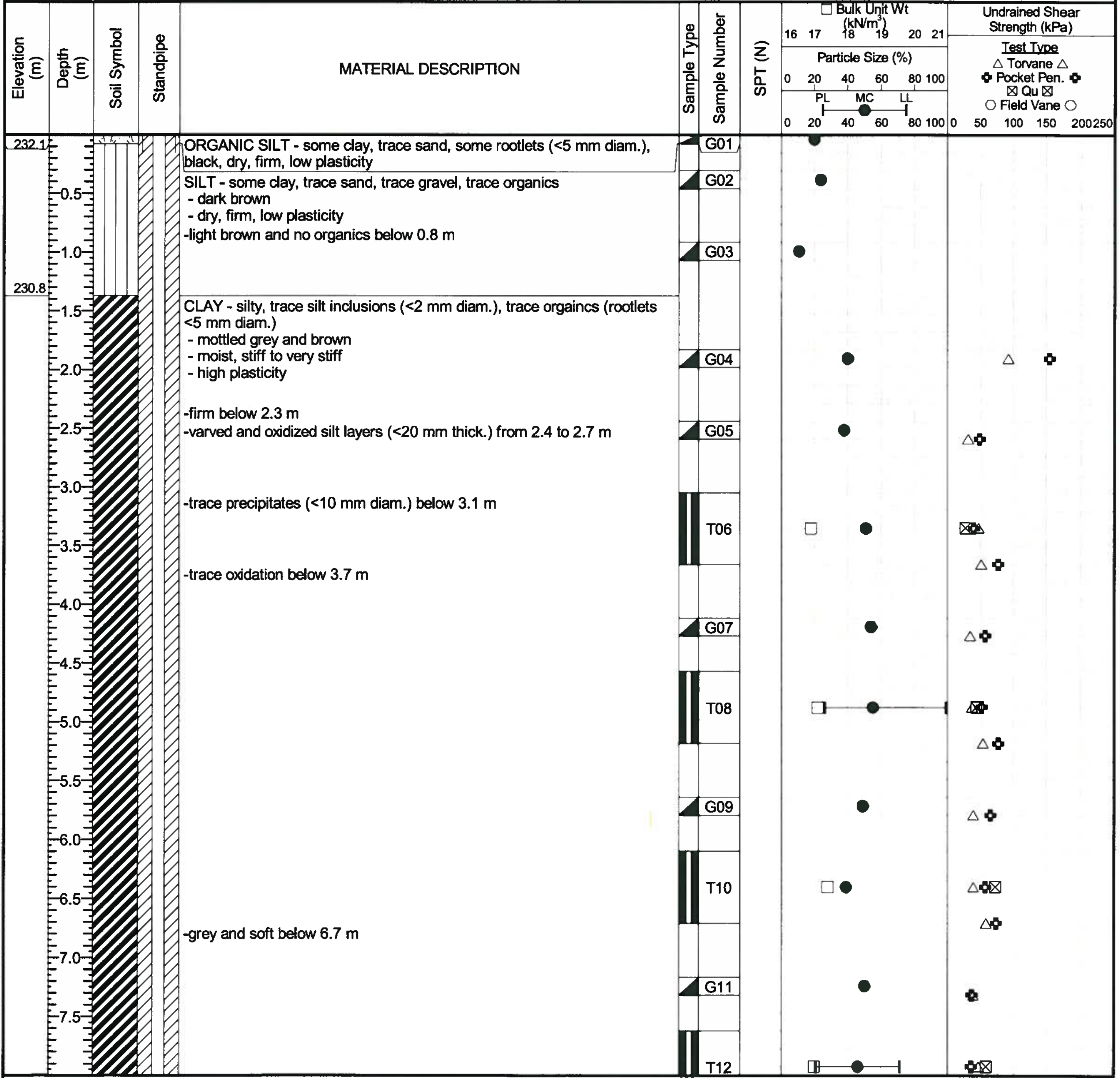
Method: 125 mm Solid Stem Auger, RM 30 Track Mount

Date Drilled: 12 July 2012

Sample Type: Grab (G) Shelby Tube (T) ☒ Split Spoon (SS) Split Barrel (SB) Core (C)

Particle Size Legend: ▨ Fines ▨ Clay ▥ Silt Sand Gravel Cobbles Boulders

Backfill Legend: Bentonite Seal ▨ Drill Cuttings Backfill Filter Pack Sand Sand at Bottom ☐ Protective Casing Slotted Pipe



ORGANIC SILT - some clay, trace sand, some rootlets (<5 mm diam.), black, dry, firm, low plasticity

SILT - some clay, trace sand, trace gravel, trace organics
- dark brown
- dry, firm, low plasticity

-light brown and no organics below 0.8 m

CLAY - silty, trace silt inclusions (<2 mm diam.), trace orgaincs (rootlets <5 mm diam.)
- mottled grey and brown
- moist, stiff to very stiff
- high plasticity

-firm below 2.3 m

-varved and oxidized silt layers (<20 mm thick.) from 2.4 to 2.7 m

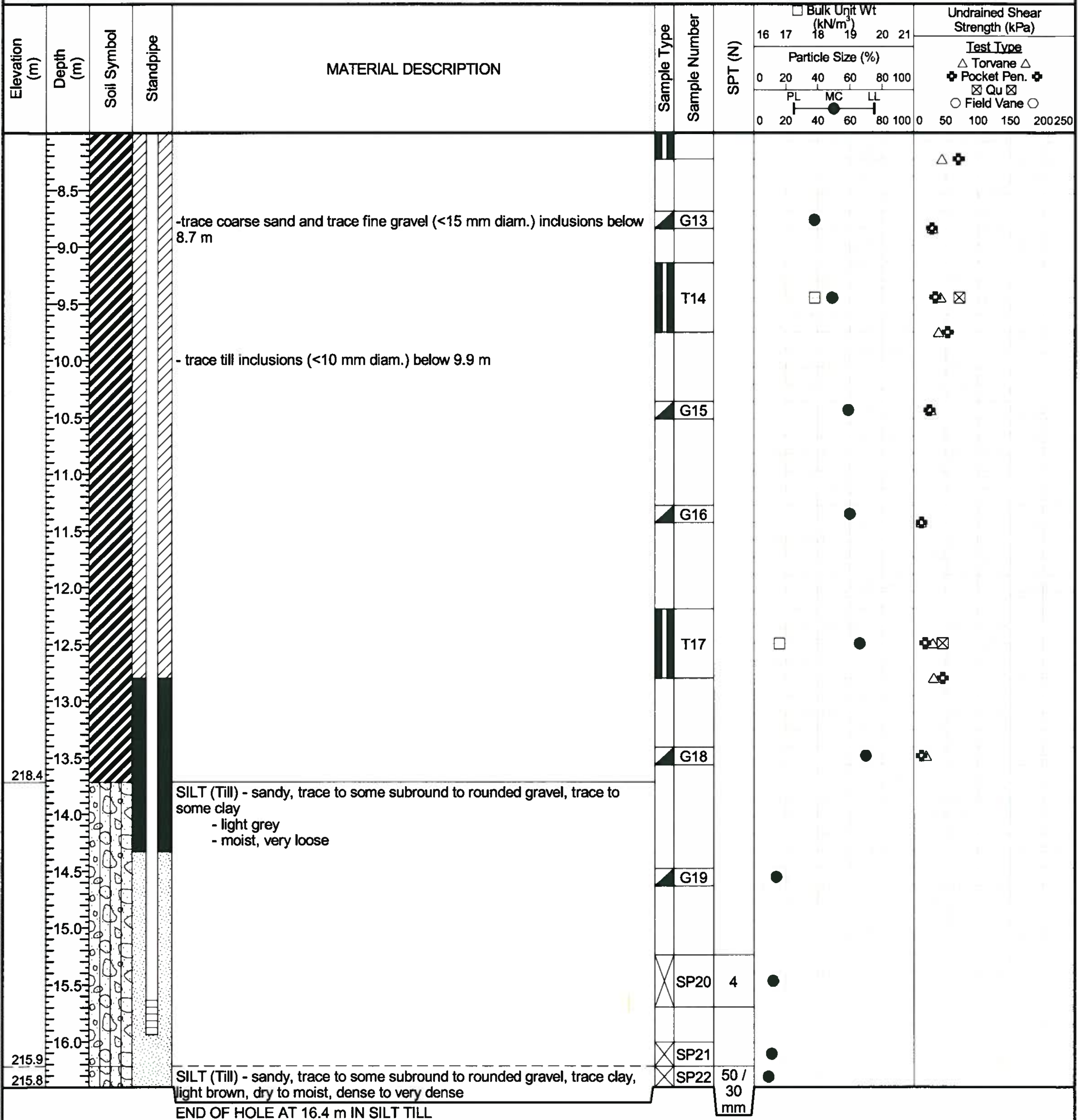
-trace precipitates (<10 mm diam.) below 3.1 m

-trace oxidation below 3.7 m

-grey and soft below 6.7 m

Logged By: Tom Hildahl **Reviewed By:** Ken Skaftfeld **Project Engineer:** Kent Bannister

Sub-Surface Log



-trace coarse sand and trace fine gravel (<15 mm diam.) inclusions below 8.7 m

- trace till inclusions (<10 mm diam.) below 9.9 m

SILT (Till) - sandy, trace to some subround to rounded gravel, trace to some clay
- light grey
- moist, very loose

SILT (Till) - sandy, trace to some subround to rounded gravel, trace clay, light brown, dry to moist, dense to very dense

Sample Number	SPT (N)
G13	
T14	
G15	
G16	
T17	
G18	
G19	
SP20	4
SP21	
SP22	50 / 30 mm

END OF HOLE AT 16.4 m IN SILT TILL
Notes:
1) Power Auger Refusal (PAR) at 16.4 m below ground surface.
2) Seepage and sloughing observed at 15.1 m below ground surface.
3) Water level was 12.3 m below ground surface immediately after drilling.
4) Stand Pipe Piezometer SP-01 installed at 15.9 m below ground surface.

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Appendix B

Photos of Existing Shoreline

Shoreline looking upstream from the Rivergate Outfall

Shoreline looking downstream from the Rivergate Outfall